POTENTIAL FLOOD HAZARDS AT WILLOW BEACH, LAKE MEAD NATIONAL RECREATION AREA

PREPARED FOR THE

NATIONAL PARK SERVICE

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POTENTIAL FLOOD HAZARDS AT WILLOW BEACH,

LAKE MEAD NATIONAL RECREATIONAL AREA

Prepared for

the

NATIONAL PARK SERVICE

by

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SUMMARY

The flood plain management guidelines of the National Park Service classify areas subject to the flash flooding as high hazard areas. Within high hazards area the hazardous flood way is that area that would be flooded during the occurence of the maximum probable flood. The guidelines specify that structures for human occupancy, structures which may offer shelter during storms, and parking, camping, or picnicking are not to be permitted in the floodway in flash flood areas.

The probable maximum flood is used regularly in the evaluation of dam safety and a general procedure for its determination has been developed. The first step is an estimate of the probable maximum precipitation (PMP) which is based on meteorological considerations. The PMP is then used to estimate the probable maximum flood (PMF) by hydrologic methods.

The National Weather Service has published a report which presents a method for calculating the PMP in the Willow Beach area. These procedures yield one-hour rainfalls which vary from 6.3 inches over Jumbo Wash to 8.8 inches for Willow Beach and Last Chance washes. Hydrologic calculations show that about 40 percent of this rainfall will runoff during the maximum hour of streamflow causing peak flows ranging from 15,000 cubic feet per second (cfs) on Last Chance Wash to 72,000 cfs on Jumbo Wash. These maximum probable flood flows would occupy the washes from one canyon wall to the other. Hence, virtually the entire area of the four canyons lies in the hazardous floodway.

Under conditions of the probable maximum flood, maximum water depths in the canyons will vary from 7 feet in Last Chance Wash to almost 24 feet in Jumbo Wash. Water velocities will range from 12 feet per second to over twenty feet per second. There would be no chance of survival for any person caught in a flood of this magnitude. Automobiles, trailers, and motorhomes would be washed down the canyons. Even permanent buildings would suffer severe damage.

In September of 1974 a flash flood swept through Eldorado Canyon on the west side of Lake Mohave killing at least nine people. The peak flow of this flood has been estimated at 76,000 cfs which suggests that it was near the probable maximum flood for Eldorado Canyon and offers support to the estimates for the several washes at Willow Beach.

POTENTIAL FLOOD HAZARDS AT WILLOW BEACH, LAKE MEAD NATIONAL RECREATION AREA

INTRODUCTION

In the four-year period, 1976-1979, five flooding incidents were reported by the rangers at Willow Beach. Each incident involved some damage to National Park Service property and associated clean-up costs. In at least one case campers lost personal property as flood waters moved through the campground. Fortunately no loss of life from flooding has been reported. However, in September 1974, a major flood swept through Eldorado Canyon causing a loss of nine lives and extensive damage to fixed property, parked cars, mobile homes, and boats moored at the at Nelson's Landing Marina. Eldorado Canyon is about 12 miles south of Willow Beach on the west shore of Lake Mohave. The National Park Service wishes to have an assessment of the flood risk at Willow Beach so that decisions regarding the continued operation of the unit can be made as required by Executive Order 11988.

This report summarizes the results of a study by Linsley, Kraeger Associates under contract to the National Park Service. The report will be part of the environmental impact statement preparatory to decisions regarding the appropriate management steps for the several washes within the Willow Beach unit of the Lake Mead National Recreational Area.

THE CATCHMENTS

The catchments under study are located at about latitude 35° 51' N, longitude 114° 37' W. They are left-bank tributaries of Lake Mohave in the state of Arizona (Fig. 1). Access Road, Willow Beach, and Last Chance Wash flow in a generally east-west direction. These three catchments are quite similar in size and general characteristics (Table 1). Jumbo Wash is, as its name implies, much larger than the other three washes and flows in a southeast-northwest direction. Data on drainage area, length, slope, and other physical characteristics are summarized in Table 1.

The area is desert with mean annual rainfall varying from about six to eight inches. Because the area is situated between elevations 600 and 4100 ft. msl, winters are relatively cool with mean January temperature about 45°F but mean July temperatures are near 90 °F. Vegetation on the basin is sparse consisting of desert grasses, sagebrush, and other low-growing vegetation. The washes all rise in the Black Mountains. These mountains are largely andesite and granite and channels are generally deeply incised. Jumbo wash includes considerable areas of volcanic rocks (tuffs and basalt).

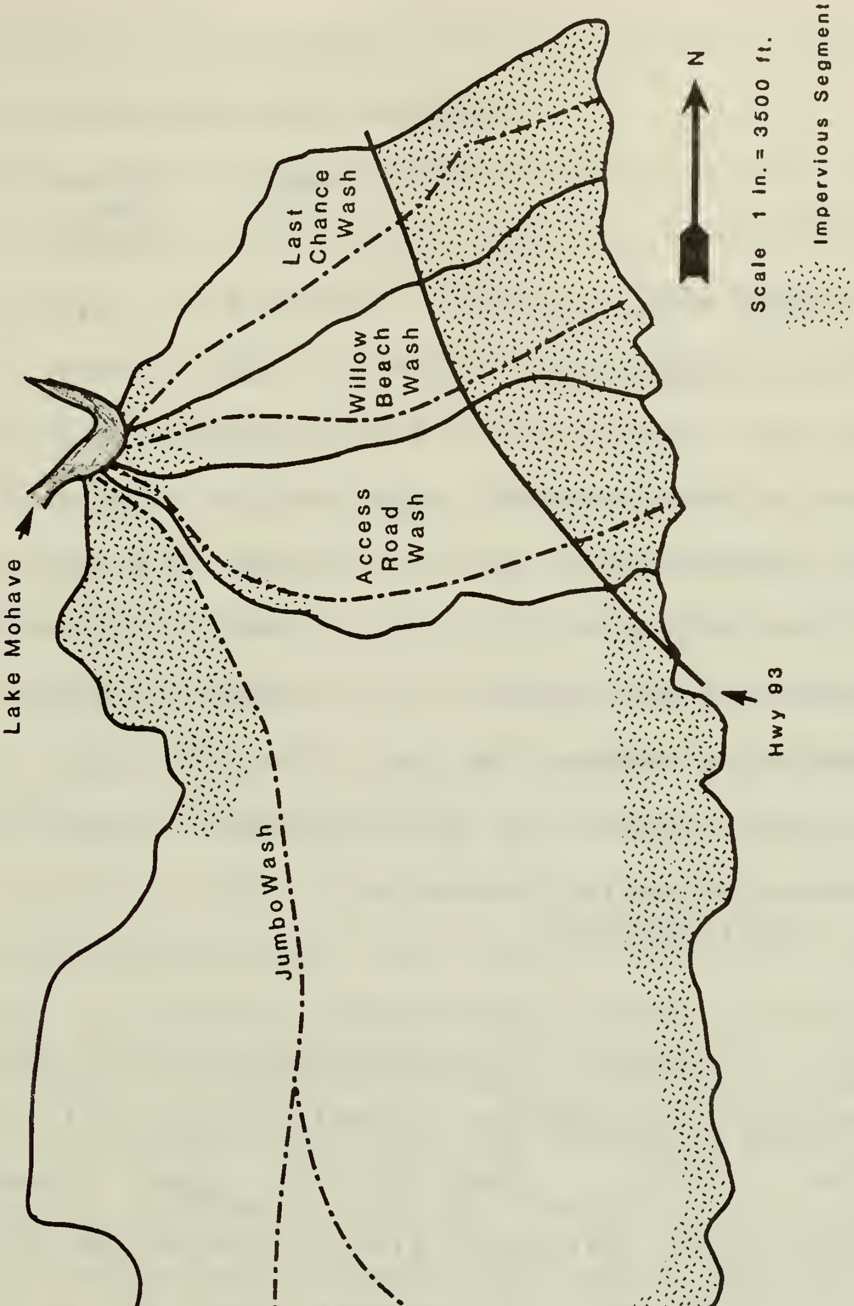


Fig. 1 Map of the Willow Beach washes

Table 1

Catchment Characteristics

	Jumbo Wash	Access Road Wash	Willow Beach Wash	Last Chance Wash
Drainage Area, sq.mi.	36.4	5.6	4.4	4.2
Channel Length, mi.	10.5	3.7	3.7	4.3
Lca, mi.	5.3	1.7	1.6	1.7
Time of Concentration, min.	92.0	32.0	32.0	38.0
Maximum Elevation, ft.	3,360	3,200	3,424	4,094
Minimum Elevation, ft.	630	630	630	630
Channel Slope, ft/mi.	260	695	755	805
Impervious Segment, sq. mi.	8.4	1.9	2.4	2.0
Pervious Segment, sq. mi.	28.0	3.7	2.0	2.2
Average Effective Impervious Area, percent	11	14	21	19

West of Highway 93 the catchments consist mostly of alluvial fans with wide shallow channels which shift position during major storms. Some of the upper channels may shift sufficiently that their flow is discharged to Lake Mohave via a different wash. For example, there is an area of about 1.35 sq. mi. between Access Road Wash and Jumbo Wash which can contribute to either wash depending on the configuration of a low divide between the washes. This could divert one-fourth of the drainage area assigned to Access Road Wash in Table 1, with a consequent reduction in flood peaks. For Jumbo Wash, an area difference of 1.35 sq. mi. is only

about four percent of the drainage area and the effect on the peak flows is correspondingly small.

Drainage areas given in Table 1 and outlined on Figure 1, were determined after careful study of topographic maps based on aerial photographs taken in 1954 and of aerial photographs taken in 1970. Channels on alluvial fans are known to shift position from time to time as a result of scour or deposition of sediment during floods, and some changes during the period 1954-70 were evident. For the smaller washes these changes can greatly affect the flood flows. Consequently we have used the largest area that might contribute to each wash, to avoid the possibility of underestimating the flood hazard.

Near Lake Mohave hard rock again dominates and the washes have cut canyons through these rocks on their way to the lake. Prior to filling of Lake Mohave, the washes entered the Colorado River at an elevation substantially lower than they do today. Hence the extreme lower end of these channels has been filled with alluvium and the channel slope has been flattened.

For the most part the exposed rock slopes are very steep (one horizontal to two vertical or more). In the alluvial fans, soils are coarse sands and gravels with most of the fines which may have once been present removed by water or

wind. Consequently these soils should be highly pervious to water, especially in the channels. Interchannel areas are armored with stones which remain after the smaller materials are washed away so that gross imperviousnes of the area may be quite high.

In connection with the use of a hydrologic simulation model described later in this report, the catchments have been divided into two segments called pervious and impervious. Figure 1 identifies the impervious segment by a dotted shading. The impervious segment includes those areas where exposed rock surfaces dominate the landscape but even in these segments there are areas of pervious alluvium. Runoff from the rock surfaces must often flow over pervious alluvium to reach a channel and during this time is subject to infiltration into the soil. Hence, the effective impervious area is less than the gross impervious area which includes all rock surfaces in the segment. From visual inspection, aerial photographs and a geologic report [1] we estimate that the gross impervious area of the impervious segment is 70 percent, of which about one-half, or 35 percent, is effective.

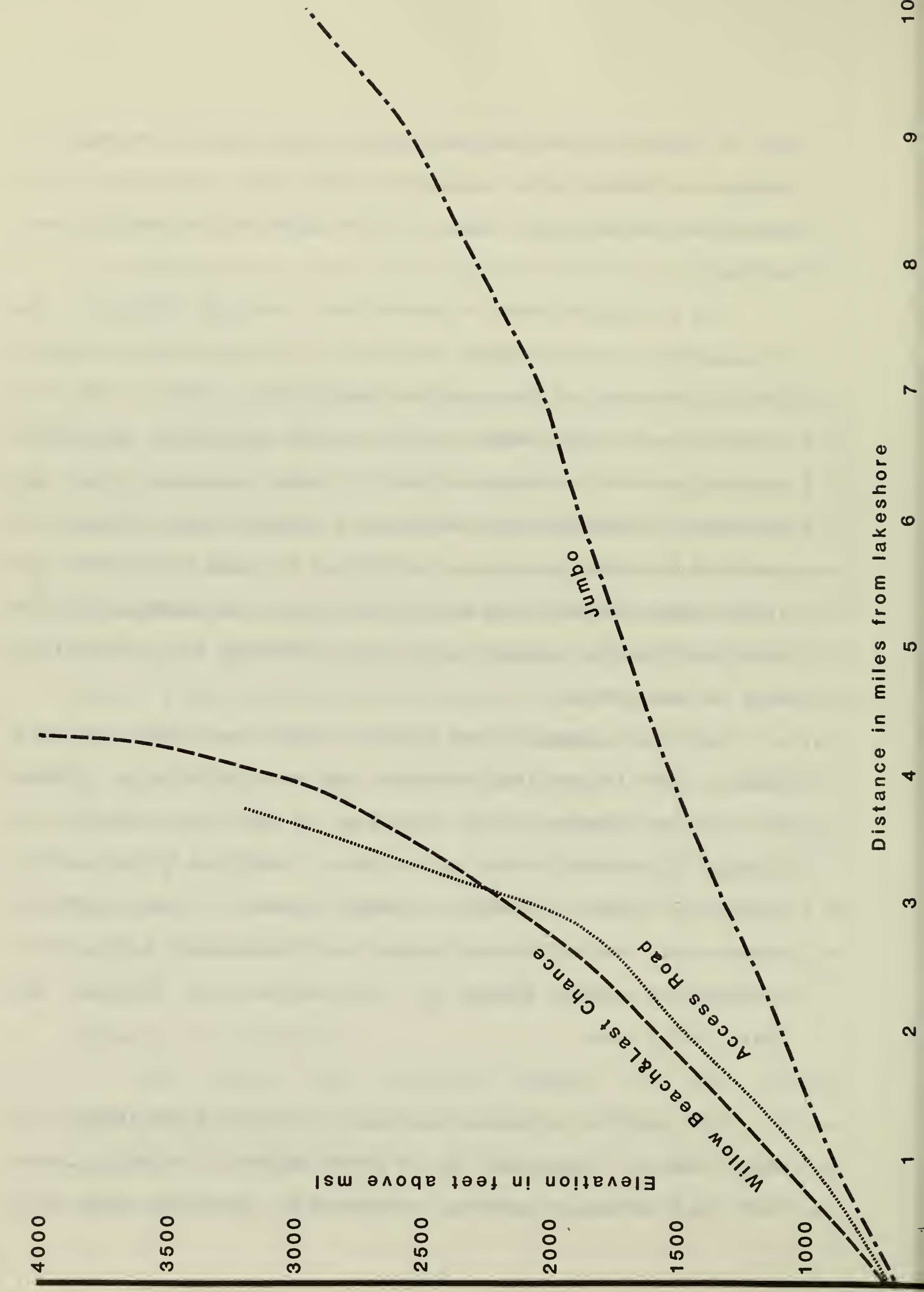
Even within the pervious segments of the several catchments, there are isolated outcrops of hard rock, and the alluvium is covered by rock fragments so that the gross impervious area of the pervious segment may also be as much

as 70 percent, but because most of the runoff from the rock fragments flows onto permeable soil, we estimate the effective impervious area of the pervious segment at four percent.

The average effective impervious area of Table 1 is calculated by multiplying the area of the impervious segment by 0.35 and that of the pervious segment by 0.04. The two products are then added and the result divided by the total area to get an average effective imperviousness for the catchment. These average values are not actually used in the analysis but they are shown in Table 1 to show that there are differences between the watersheds with, for example, Willow Beach Wash having nearly twice the percent of the impervious area of Jumbo Wash.

As the washes narrow prior to their entrance into Lake Mohave, flow is confined between rock walls with side slopes of 1:2 or steeper and little or no soil cover except for locally flat areas on the ridge tops. Profiles of the washes (Fig. 2) show extremely steep slopes in the mountain headwaters, much flatter slopes in the alluvial fans, and a tendency to steepen slopes as the washes cut through the lower rocky zone.

The profile of the main channel of Jumbo Wash (Fig.2) is much flatter than that of the other washes. However, Jumbo Wash has steeply sloping tributaries entering the main



Elevation in feet above msl
4000
3500
3000
2500
2000
1500
1000
Distance in miles from lakeshore
1
2
3
4
5
6
7
8
9
10
Jumbo
Access Road
Willow Beach&Last Chance

channel throughout most of its length. Jumbo Wash has a dendritic channel pattern while the other washes have a more fan-like pattern.

Because of the steep slopes and relatively short channel lengths the time of concentration of the catchments is quite short. Time of concentration is defined as the time required for water to flow from the most remote point of the catchment to the outlet. If we assume an average water velocity of 10 feet per second, a conservative estimate for the PMF (Table 9), we can calculate the concentration times given in Table 1 for the channel lengths as shown. The shorter washes have a time of concentration slightly over one-half hour while Jumbo wash with its greater length shows a time of concentration of about 1.5 hours. The time from beginning of heavy rain to peak is likely to be less than the time of concentration. If a storm were centered over the lower portion of the catchment the peak would occur at about the same time as the end of the heavy rain. Thus for heavy rains lasting less than 30 minutes, the time to peak in the three smaller washes will be about equal to the duration of rain. Time of concentration will generally be smaller for the greater floods because of the higher flow velocities which occur in such large floods. Table 1 also gives values of Lca, the length to center of area of the catchment. This is the distance from the outlet of the catchment to the centroid of the basin. For the three smaller washes Lca is less than half of the channel length,

indicating that the catchment area tends to be concentrated downstream of the basin midpoint.

GENERAL HYDROLOGY

Rainfall occurs in the Willow Beach area during two seasons. Winter rain results from the passage of Pacific frontal systems which have moved through California and Nevada. These fronts bring rain of moderate intensity which may last for several hours, and a series of frontal passages may continue for several days. Runoff rates are usually low but runoff may persist for several days and major floods may occur. Occasionally during the winter months a tropical storm from the Pacific Ocean west of Mexico or a cold front may penetrate, bringing intense rains capable of causing flash floods. Usually however the most intense rains and those most likely to cause flash floods occur in the summer and early fall (July through October) with the most critical month being September. Moist maritime air from the Pacific enters the area when the circulation is favorable and convective thunderstorms occur when the combination of air mass stability and local topography are appropriate. It is therefore possible for serious flooding to occur in any month of the year.

Thunderstorms may remain at or near the point where they form or they may drift with the local wind. These winds are

most frequently from the western quadrant (NW to SW winds) but movement in any direction is possible, depending on the then existing atmospheric circulation patterns. Thunderstorms associated with a cold front passage will move with the front. Thunderstorms may vary in areal extent from a fraction of a square mile to 25 square miles or more. Combinations of multiple cells may cause rain over relatively large areas.

ANALYTIC METHODS

Desert washes such as the four washes at Willow Beach pose difficult analytical problems for the hydrologist. No record of flow has been maintained on any of these washes. The closest long-term rainfall record is at Boulder City, Nev., about 13 miles northwest. The nearest recording gage record is at Searchlight, Nev., about 34 miles to the southwest. A short record exists at the Willow Beach NPS residential area but a brief comparison shows that it does not accurately indicate the rain over all of the washes because of the characteristically spotty rain during thunderstorms. These conditions preclude the classical approach of hydrology, namely to devise some relationship between local rainfall and local runoff.

Two general type approaches are available:

(1) To use some type of statistical procedure based on the streamflow records available in the region. This is called regional flood frequency analysis.

(2) To assume that rainfall characteristics from some nearby location are applicable to Willow Beach and that the resulting flood flows can be computed by some appropriate hydrologic procedure.

Several different methods are possible within these two general approaches. Any method that is used will have some degree of uncertainty, because these methods depend on assumptions which may not be completely accurate. We have used several methods and compared the results from these methods in an attempt to derive a "concensus answer" which may be superior to any one of the answers from a particular method.

The computed peak flows by all methods are summarized in Table 7 and plotted as flood frequency curves for Access Road Wash in Fig. 3. The following sections describe each of the methods utilized and discuss the strengths and weaknesses of each method.

Regional Flood Frequency Methods

A report dealing with flood frequency in Arizona was

prepared by the U.S. Geological Survey for the Arizona Department of Transportation [2]. This method is designated GSADOT for reference in this report.

This method utilized available streamflow records in Arizona. Each record was analyzed to define the 2-, 5-, 10-, 50-, 100-and 500-year floods at the station. Values of the floods for a particular return period from a number of stations were then correlated with parameters describing the catchments. The regression equation giving the best conformance with observed data was adopted. In this case the only significant parameter was the drainage area and the resulting equations are exponential (Table 2).

Equations were developed for subregions of the state. The Willow Beach area falls within the northwest region. Unfortunately there are very few streamflow stations close to Willow Beach. There are three stations in the Kingman, Ariz. area, but most of the stations used for the northwest Arizona region are located in the upper Gila basin near Flagstaff and the central Little Colorado basin near Page, Ariz. Significant geological and climatic differences exist between these stations and Willow Beach.

Table 2

Flood Frequency Equations for Northwest Arizona

Arizona Department of Transportation Study

Return Period yr	Values of K and a in $Q=KA^a$ K	a
2	19.0	0.660
5	66.3	0.600
10	127.	0.566
25	252.	0.532
50	393.	0.510
100	584.	0.490
500	1,300.	0.451

This approach makes two critical assumptions:

(1) That all catchments within a region are similar except for the differences in drainage area. Actually soil type, topography, vegetation, elevation, and precipitation differences could be important.

(2) That the relatively short records of flow available correctly define the flood peaks at the several stations for the various return periods. Generally short records are unreliable for frequency analysis. The estimates are more likely to be high than low.

Rainfall-Runoff Models

Two different approaches using a rainfall-runoff model were tried. The model employed was a version of the Stanford Watershed Model (SWM) [3]. This model uses rainfall data as input and simulates interception, impervious area runoff, infiltration, interflow, groundwater, channel routing and the

other processes of the runoff cycle. The model is adjusted to the catchment under study by setting a number of parameters to accord with catchment characteristics. If possible the model is tested by simulating flows for a period when measured flows are available and the parameters calibrated by trial until the observed flows are closely simulated. In this case no measured flows were available. However, rainfall data from Willow Beach concurrent with the several flooding incidents were available, and an approximate calibration was effected using these data.

The parameters selected for the watersheds are listed in Table 3 for three trials, A, B, and C. They have been arranged in two groups. Group one are the parameters which dominate the simulation process under desert conditions. These parameters are the percentage impervious area, the upper and lower zone moisture storage capacity, and the infiltration and interflow parameters. These parameters were tested on the several data sets. Group two are those parameters which are significant and for which values have been assigned from map analysis or field inspection. These include the interception storage,and the length, slope and roughness of overland flow. Finally there are several parameters which are not relevant in this study. No interflow or groundwater flow is assumed to occur and hence the interflow and groundwater recession factors have no

significance. Calculations are made over a period of a few hours and evaporation and evapotranspiration are not significant. Finally it is assumed that we are dealing with true rainfall and the rainfall correction factor is not relevant. Values for this group of parameters are not included in Table 3.

TABLE 3

SWM Parameters

Parameter		Segment 1 A	Segment 1 B	Segment 1 C	Segment 2 A	Segment 2 B	Segment 2 C
KEY PARAMETERS							
A	Impervious fraction	0.7	0.35	0.35	0.04	0.04	0.04
UZSN	Upper zone, in.	0.1	0.10	0.25	0.25	0.25	0.25
LZSN	Lower zone, in.	2.0	2.0	6.0	6.0	6.0	6.0
INFIL	Infiltration, in/hr	0.25	0.25	0.44	0.44	0.44	0.44
INTER	Interflow fraction	0	0	0	0	0	0
PHYSICAL PARAMETERS							
EPXM	Interception	0.05	0.05	0.05	0.05	0.05	0.05
L	Overland flow length,ft.	75.	75.	75.	75.	1000.	1000.
SS	Overland flow slope, ft/ft	0.1	0.1	0.1	0.1	0.1	0.1
NN	Overland roughness	0.3	0.3	0.3	0.3	0.3	0.3

Segment 1 represents the hard-rock areas and is called the "impervious segment". Segment 2 represents the alluvium and is the "pervious" segment. (Table 1 and Figure 1).

Using the reported rainfall from the station at Willow Beach, the streamflows for the Access Road wash were simulated. There were no measured stream flows, but we have estimated in Access Wash from the rangers description (Table

4) flows to have been in the order of 300 cfs. Parameter set A yielded flows which appeared to be much too high for the relatively minor events which occurred. This parameter set also produced a Probable Maximum Flood event much higher than could be normally expected. Hence, set A was not given further consideration. Parameter sets B and C gave almost identical flows for the lesser floods and these flows appeared to be reasonably consistent with the ranger's description of the incident. At the low flow range the simulated flows were reasonably consistent with the 1-and 2-yr floods from GSADOT. Hence, parameter sets B and C are considered plausible sets for the Willow Beach washes.

Table 4

Calibration Data

Accesss Wash Road

Date	Rainfall	Reported Depth	Estimated Q	Simulated Q A	B	C
9/10-11/76	2.75	3-4 ft	400	239	114	113
9/10/77	1.96	none reported	-	400	220	220
8/12/78	0.87	3-4 inches	50	165	70	70
9/13/78	0.86	2-2 1/2 ft.	250	243	120	120
3/28/79	1.11	8-10 inches	100	310	144	144

The model was used to generate flood series for each of the four washes using two sets of rainfall data. The first set was from NOAA Atlas 2, Precipitation Frequency Atlas for the Western States [4]. This atlas provides estimates of the 24-hr rainfall for return periods of 2-, 5-, 10-, 25-, 50-,

and 100-yr, and procedures to reduce this rainfall to durations from 5 minutes to 12 hours. These procedures were used to develop a 24-hr rainfall sequence for each return period. The atlas also includes procedures for adjusting the rainfall amounts for the size of the drainage area. The three small catchments are so nearly equal in size that the same rainfall was used for all three catchments (Table 5). Rainfall depths for the larger area of Jumbo Wash are slightly lower (Table 6). The maximum 6 hours of these rainfall sequences was used as input to the simulation model. Runs were made using both parameter sets B and C.

Table 5

Rainfall Intensities for Small Catchments, inches

Return Period	Duration 5m	10m	15m	30m	1h	2h	3h	6h	12h	24h
2	0.27	0.45	0.62	0.74	0.93	1.02	1.10	1.20	1.30	1.40
5	0.33	0.51	0.65	0.90	1.16	1.40	1.50	1.70	1.80	2.00
10	0.39	0.60	0.76	1.00	1.36	1.60	1.70	2.00	2.20	2.40
25	0.46	0.72	0.92	1.20	1.65	2.00	2.10	2.40	2.70	3.00
50	0.55	0.86	1.04	1.50	2.00	2.22	2.40	2.80	3.10	3.40
100	0.68	1.04	1.30	1.90	2.40	3.00	3.10	3.20	3.40	3.80

Table 6

Rainfall Intensities for Jumbo Wash, inches

Return Period	Duration 5m	10m	15m	30m	1h	2h	3h	6h	12h	24h
2	0.24	0.36	0.47	0.69	0.88	0.94	1.00	1.20	1.20	1.40
5	0.33	0.51	0.65	0.90	1.16	1.40	1.50	1.70	1.80	2.00
10	0.39	0.60	0.76	1.00	1.36	1.60	1.70	2.00	2.20	2.40
25	0.46	0.72	0.92	1.20	1.65	2.00	2.10	2.40	2.70	3.00
50	0.55	0.86	1.04	1.50	2.00	2.20	2.40	2.80	3.10	3.40
100	0.68	1.04	1.30	1.90	2.40	3.00	3.10	3.20	3.40	3.80

The second source of rainfall data was from the National Weather Service recording rain gage at Searchlight, Nevada which is about 34 miles southwest of Willow Beach. A 25-yr record of hourly rainfall was developed from the available data for the period 1952 to 1976. This 25-yr record was input to the simulation model, annual peaks for each year were determined, and the resulting data series analyzed for flood frequency.

The principal assumptions involved in this approach to the development of flood frequency are:

(1) That the n-year rainfall will produce the n-year flood. This assumption is questionable in many areas because of the variable conditions which may precede the storm. However, given the high evaporation and the relative infrequency of rainfall in the southwestern desert region, it seems reasonable to assume that all rainfalls occur on uniformly dry soil. This assumption leads to the smallest flood flows that could be expected from a given rainfall.

(2) That it is possible to arrive at reasonable assumptions as to the parameters of the rainfall-runoff model. Because of the peculiar character of these desert catchments, it appears that the most important parameter is the effective impervious area. Our simple calibration seems to indicate that the selected values for impervious area are reasonable.

(3) That the rainfall recorded at Searchlight, Nev. is representative of rainfall that might have occurred over any of the Willow Beach washes. Given the close proximity of Searchlight to Willow Beach and the fact that both locations seem to have about the same mean annual precipitation it is believed that this is a valid assumption.

COMPARISON AND ASSESSMENT OF THE FLOOD FREQUENCY ESTIMATES

The results of flood estimates by each of the methods described above are tabulated for all washes in Table 7.

Table 7

Calculated Peak Flows For Various Return Periods, cfs

Return Period	2	5	10	25	50	100
From Geological Survey Arizona Study (GSADOT)						
Access Road	59	186	337	630	946	1358
Willow Beach	51	161	293	554	837	1207
Last Chance	49	157	286	541	817	1180
Jumbo	204	573	971	1706	2458	3399
From NOAA Precipitation Data with Parameter Set C (NSW-SWM C)						
Access Road	336	484	563	754	975	1195
Willow Beach	395	590	688	887	1141	1439
Last Chance	348	510	595	786	969	1282
Jumbo	474	1110	1480	2054	2370	2685
From NOAA Precipitation Data with Parameter Set B (NSW-SWM B)						
Access Road	336	541	707	1214	2259	3670
Willow Beach	396	644	848	1402	2235	2770
Last Chance	348	538	754	1180	1873	2373
Jumbo	727	1080	1980	2660	4025	6544
From Searchlight Rainfall Data - Parameter Set C (NWS-SWM C)						
Access Road	207	408	578	727	853	979
Willow Beach	340	459	622	869	1027	1177
Last Chance	150	400	562	822	980	1138
Jumbo	474	1110	1420	2054	2370	2685

The flow frequency curves for Access Road Wash are shown for all methods in Fig. 3. Our analysis of these results is as follows:

(1) The Searchlight data yields lower peaks at all return periods than do either of the other rainfall-runoff methods (NWS-SWM B and NWS-SWM C). This situation seems to result from a fundamental problem in the use of published hourly rainfall data for small watersheds which respond to very short duration, high intensity rain. Since the hourly data is published on a clock-hour basis, any rainfall which extends across a clock-hour is distributed to two hours. Thus, one inch of rainfall occurring between 3:45 and 4:45 at an actual intensity of two inches per hour, is published as two one-hour amounts of 0.50 inches. This causes the calculated peak to be much lower than it should be. Even if the 30-minute rainfall had occurred entirely within a clock-hour, the apparent intensity is one-half of the actual intensity. Consequently, the flood peaks computed from the Searchlight data are probably low.

(2) The two methods using National Weather Service rainfall frequency data are mutually exclusive since they assume different parameters for the rainfall-runoff model. The difference between the two curves is caused by the differing assumptions as to UZSN, LZSN, INFILTRATION and L in Segment 1. Assuming that we have correctly indentified the impervious fraction in Segment 1, there is no reason to

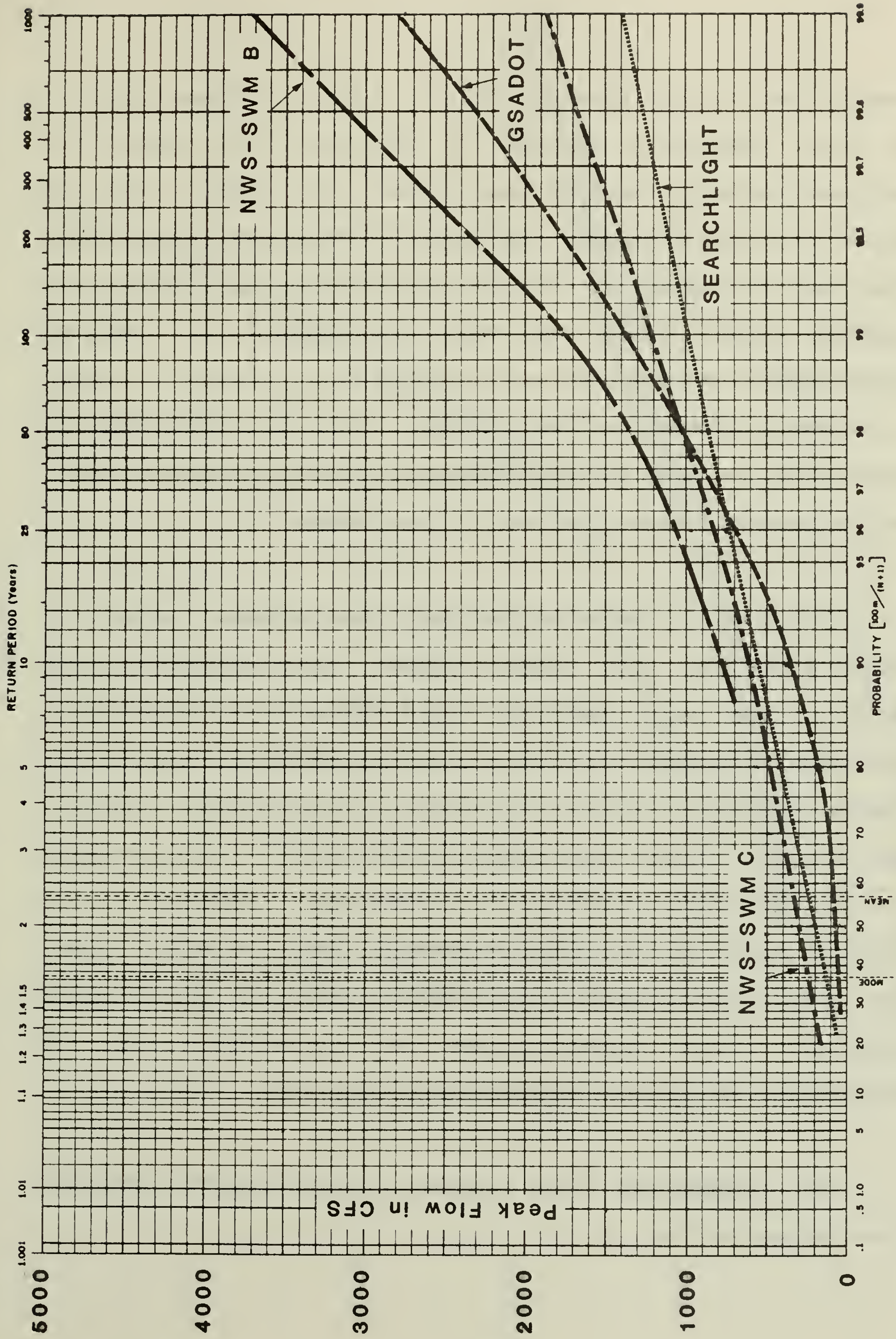


Fig. 3 Comparison of results by different methods for Access Road Wash

assume that the hydrologic behavior of the pervious portion of this impervious segment is different than that of the pervious segment. Because land slopes are steeper in the areas of rock exposure, the length of overland flow is much less than in Segment 2. A long overland flow path means that surface runoff which may occur is exposed to delayed infiltration during the time required for the water to reach a channel. However, values of UZSN, LZSN and INFILTRATION should be the same in both segments. Hence, parameter set C is the logical choice for use in this study.

(3) Flows estimated by NWS-SWM C are somewhat higher for Willow Beach and Last Chance Washes than estimated from GSADOT. For Jumbo and Access Road Washes the lesser floods are somewhat higher than those given by GSADOT but the 100-yr floods are lower. Flows estimated from Searchlight rainfall are slightly lower than GSADOT or NWS-SWM C. These results are, by hydrologic standards, in reasonable agreement, and this can be construed as the concensus answer we seek. Since the simulation approach (NWS-SWM C) generally gives higher flows for most return periods on all washes, it is recommended as the more conservative estimate.

The recommended flood frequency curves based on NWS-SWM C are shown in Fig. 4.

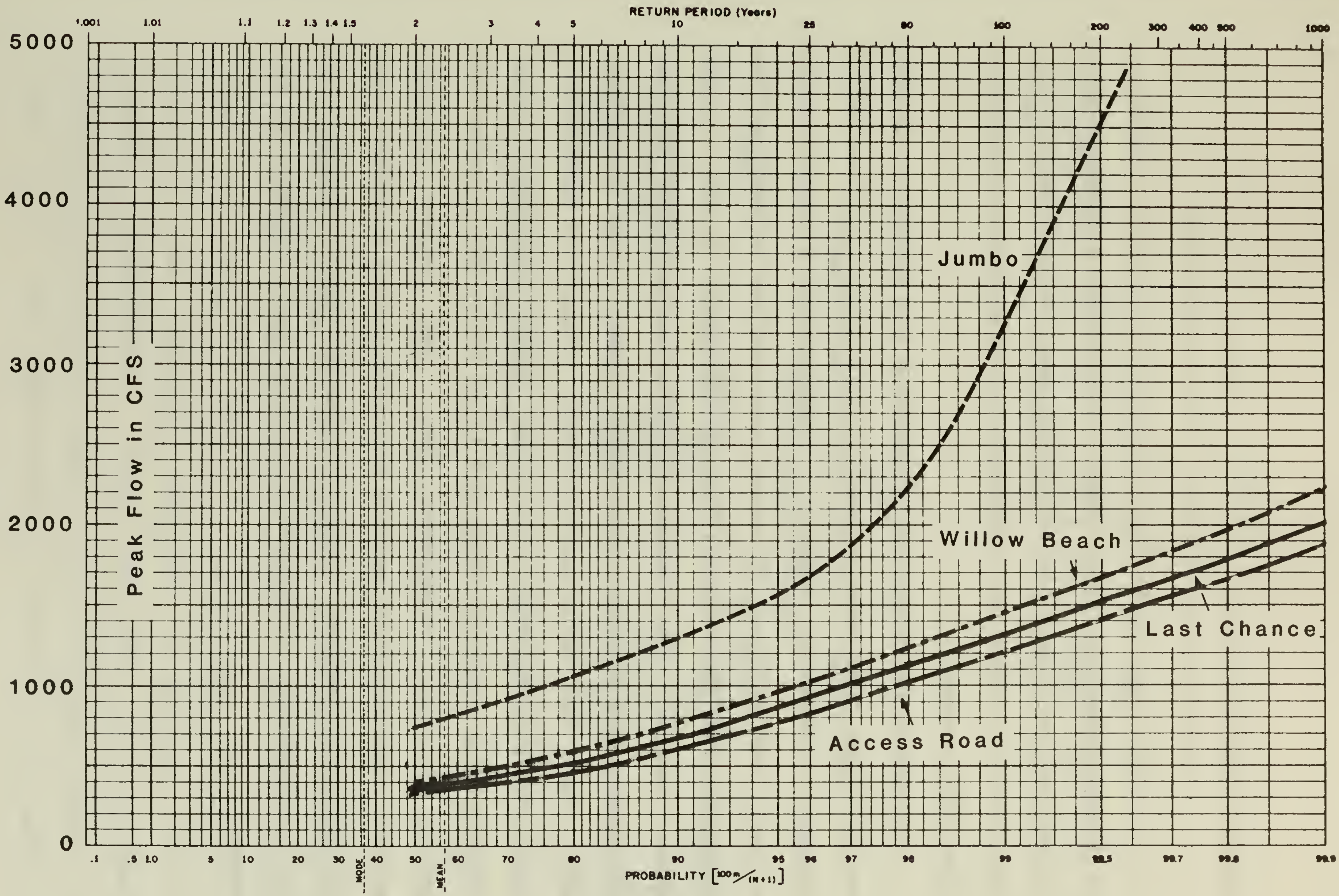


Fig. 4 Flood Frequency Curves for the Willow Beach Washes

PROBABLE MAXIMUM FLOOD

A widely used basis for evaluating the safety of dams is the Probable Maximum Flood (PMF). This flood is specified in the National Park Service Flood Management Guidelines as the flood to be used in determining the hazardous floodway in flash flood areas.

The usual procedure is to estimate the Probable Maximum Precipitation (PMP) and to convert this storm into the PMF by suitable hydrologic methods. The National Weather Service has published a report [5] which provides the methodology for estimating the PMP rainfall for any catchment in the western United States. The procedure enables estimates of both the PMP Convergence Rainfall which might result from a general storm over the area and a Local Storm PMP caused by a convective thunderstorm. For the small catchments under consideration in this study the Local Storm PMP is much more severe than the convergence PMP. The derived rainfall amounts and durations up to 6 hours are given in Table 7 for the individual washes. Note that the 15 min. rainfall of the PMP equals or exceeds the 100-yr 24-hr rainfall in all washes!

Table 8

PMP Rainfall for the Willow Beach Washes

Wash	Duration								
	15m	30m	45m	1h	2h	3h	4h	5h	6h
Access Road	5.9	7.3	8.0	8.5	9.9	10.6	11.0	11.4	11.6
Willow Beach	6.1	7.6	8.3	8.8	10.2	10.9	11.3	11.7	11.9
Last Chance	6.1	7.6	8.3	8.8	10.2	10.9	11.3	11.7	11.9
Jumbo	3.7	5.1	5.7	6.3	7.5	8.2	8.8	9.2	9.8

Table 9

Probable Maximum and 500-year Flood Peaks,

Willow Beach Washes

Wash	Peak Flows		
	500-yr flood	Probable Maximum Flood	
	cfs	cfs	cfs/sq mi
Access Road	1650	17,500	3120
Willow Beach	1980	16,000	3640
Last Chance	1800	15,000	3570
Jumbo	6100	71,800	1970
Eldorado	----	76,000*	3210*

* Estimate of peak of flood of September 14,1974 by U.S. Geological Survey [6]

We have transformed the PMP rainfall into runoff hydrographs using the SWM system and parameter set C. The resulting peak flows are given in Table 9. The PMF peaks illustrate the fact that with sufficient rainfall to satisfy the soil moisture deficiencies which are usual in the desert, extraordinary floods can result. The PMF floods range from about 11 to 26 times the 100-yr flood. It is a matter of

interest that the computed Probable Maximum Flood flows for the Willow Beach Washes are roughly equal to the estimated peak flow during the Eldorado Canyon flood [6] of September 1974 when expressed in cubic feet per second per square mile. It would be expected that the smaller catchments at Willow Beach would have higher flows per unit area. Thus the Eldorado Canyon event seems to have approached the probable maximum flood for the catchment.

Since the 500-yr flood sometimes enters decision-making regarding flood problems, values of the estimated 500-yr peaks are also included in Table 9. No estimates of the 500-yr rainfall intensities are available. Hence, we have estimated the flood peaks from the frequency curves of Figure 4.

SEDIMENT AND DEBRIS

Each of the incident reports from Willow Beach notes that considerable amounts of sediment were brought down by each runoff event. Sediment transport typically varies as a power of the flow rate. Thus one may expect that the occurence of the lower frequency floods (25 to 100 yr) will transport vastly more sediment and debris than has ever been experienced in the few years that the Willow Beach facility has been operating. There are many equations for predicting

the movement of bed sediment. Answers from these equations can differ by more than an order of magnitude when applied in a relatively simple case. In the desert washes, the location of the storm center, the formation of temporary gravel dams, obstacles in the flow, variable slope and flow rate all contribute to very large uncertainty as to the sediment transport to be expected.

Any attempt to accurately compute the actual sediment load to be expected in these washes during a specific flood would be futile. It is estimated that the 100-yr flow will transport at least 20 to 40 times as much sediment as any of the minor floods experienced in the area since the Willow Beach facility has been operating. Most of the sediment will be deposited near the lakeside where the slope is quite flat and much will deposit wherever obstacles (buildings, shrubs, walls, etc.) create a local area of low velocity, or where excavations create a sediment trap.

During a probable maximum flood, sediment transport will be very large, probably amounting to ten to twenty acre-feet for the smaller washes and perhaps as much as one hundred acre-feet from Jumbo Wash. Much of this will be carried to and deposited in the lake.

FLOOD PLAIN BOUNDARIES

The depth of the flood at each of several cross-sections

was calculated using the critical-depth for each flow rate at the cross-section. The calculations ignore the effect of major obstructions such as mobile homes and buildings because it is difficult to estimate the effect of such obstructions and because it is uncertain whether the obstruction will remain in place. It is certain, however that any obstructions in the flow will cause somewhat greater flood depth than those calculated.

We selected the critical depth calculation in lieu of using backwater computations because of the absence of a suitable control section from which to begin the computation and the difficulties of dealing in a reasonable way with sediment, scour, obstructions and the dynamic effects of the flood waves. The effect of bulking of the flow with sediment has also been ignored. The quantity of sediment transported under normal flow conditions is usually not sufficient to cause much bulking. However, in desert washes, waves of sediment sometimes form a temporary dam which is eventually overtopped causing a surge of water and sediment to move downstream. Such conditions are virtually unpredictable but, as with obstructions, the effect is always to increase the flood depth. Thus the computed depth given in Table 10 should be viewed as only minimum likely depths with the prospect that actual elevations will be somewhat higher.

The channels of the several washes have been excavated to increase capacity. As they approach the lake, excavation

requires a flattening of the channel slope which in turn encourages sediment deposition. The channels near the lake can be expected to fill with sediment early in a flood so that the peak will probably occur with the channel partially filled and with flood levels higher than those estimated in this report.

Plates 1 through 4 present cross-sections and plan maps showing estimated elevations for the various frequencies in the downstream section of each wash. Table 11 gives the mean and maximum depth and the mean velocity at each section.

Since the occupied portions of the facility are generally in the canyons where the washes cut through the rock exposures near the lake, the significance of the flood plain mapping can be stated as follows:

> Floods with return periods of 5- to 25-years generally exceed the channel capacity at some point in each wash and begin to spread out across the canyon floor. This would be hazardous for children and physically handicapped, or campers caught by surprise during the night.
>
> Floods with return periods of 50- to 100-years will flood the canyons from cliff to cliff at one or more sections in the wash. On Access Road Wash this condition can develop with the 5-yr flood or less.
>
> The 500-yr event floods the canyons from wall to wall at flow depths ranging from 1.8' - 3.0' on Access Road Wash, 1.8' - 6.3' on Willow Beach Wash, 1.8' - 4.9' on Last Chance Wash and 3.0' - 8.2' on Jumbo Wash
>
> The probable maximum flood will fill the canyons from wall to wall to depths of 15 to 20 ft. Chances of survival would be near zero.

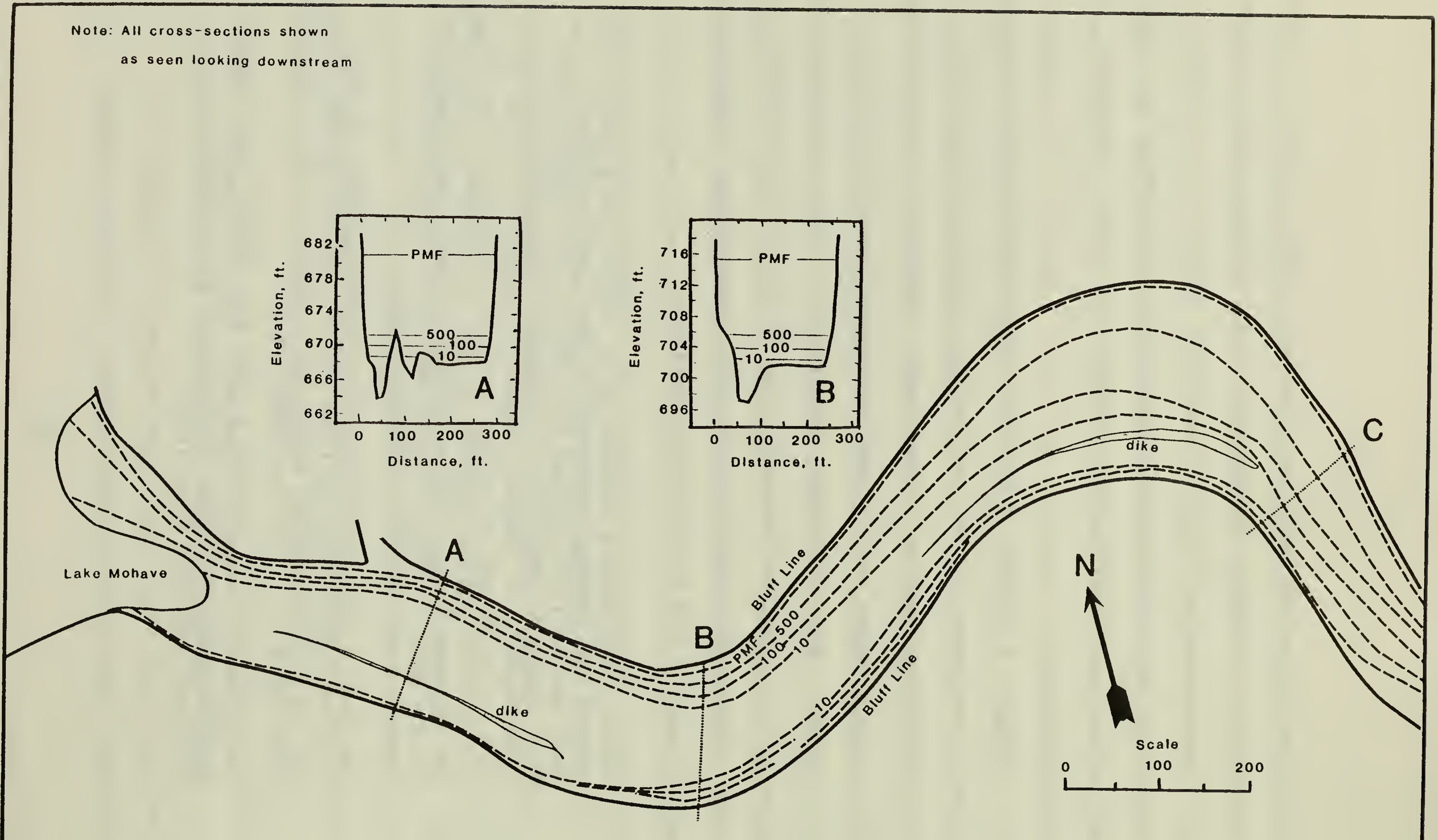


Plate 1. Flood Plain Limits in Jumbo Wash

(Part 1)

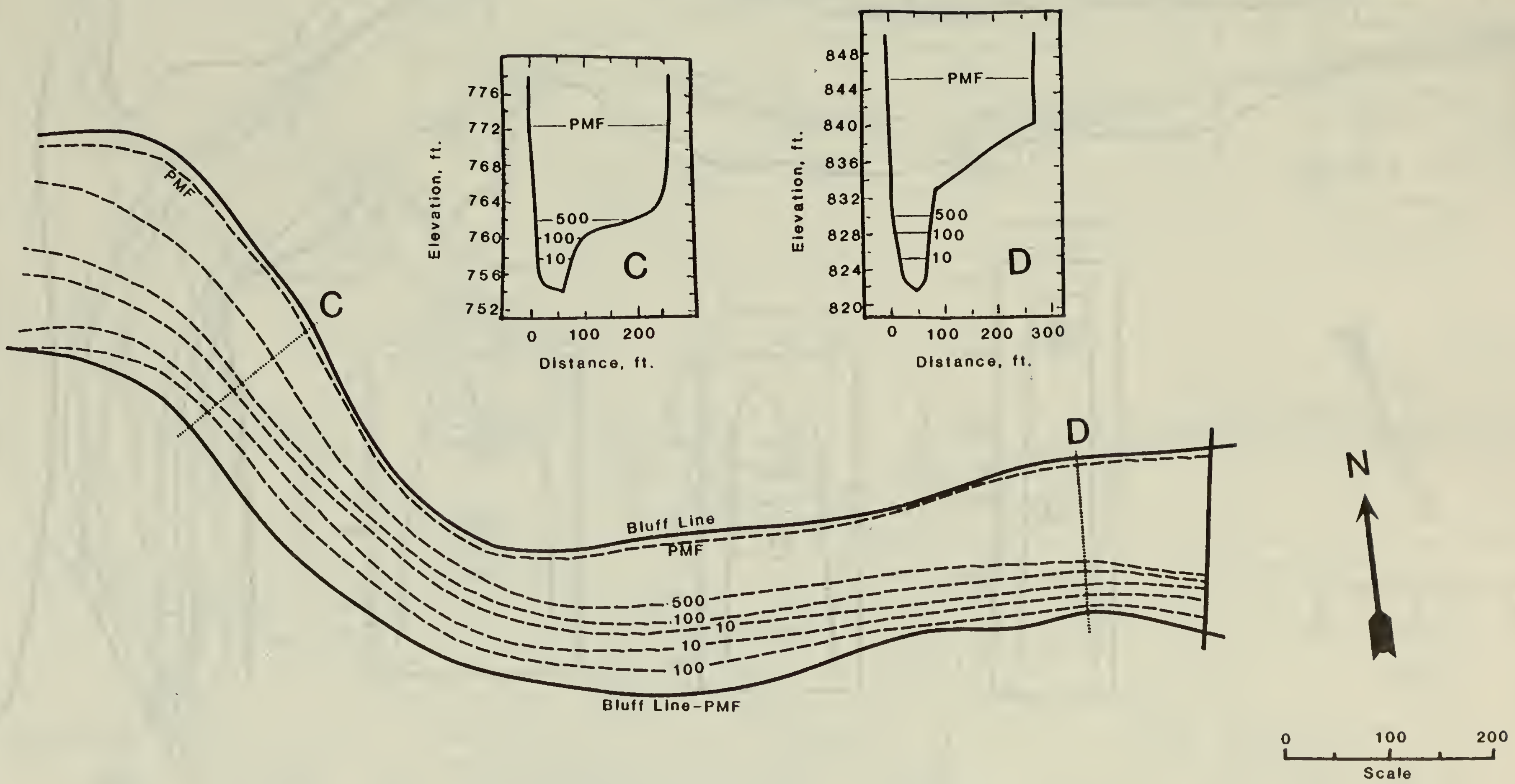


Plate 1. Flood Plain Limits in Jumbo Wash (Part 2)

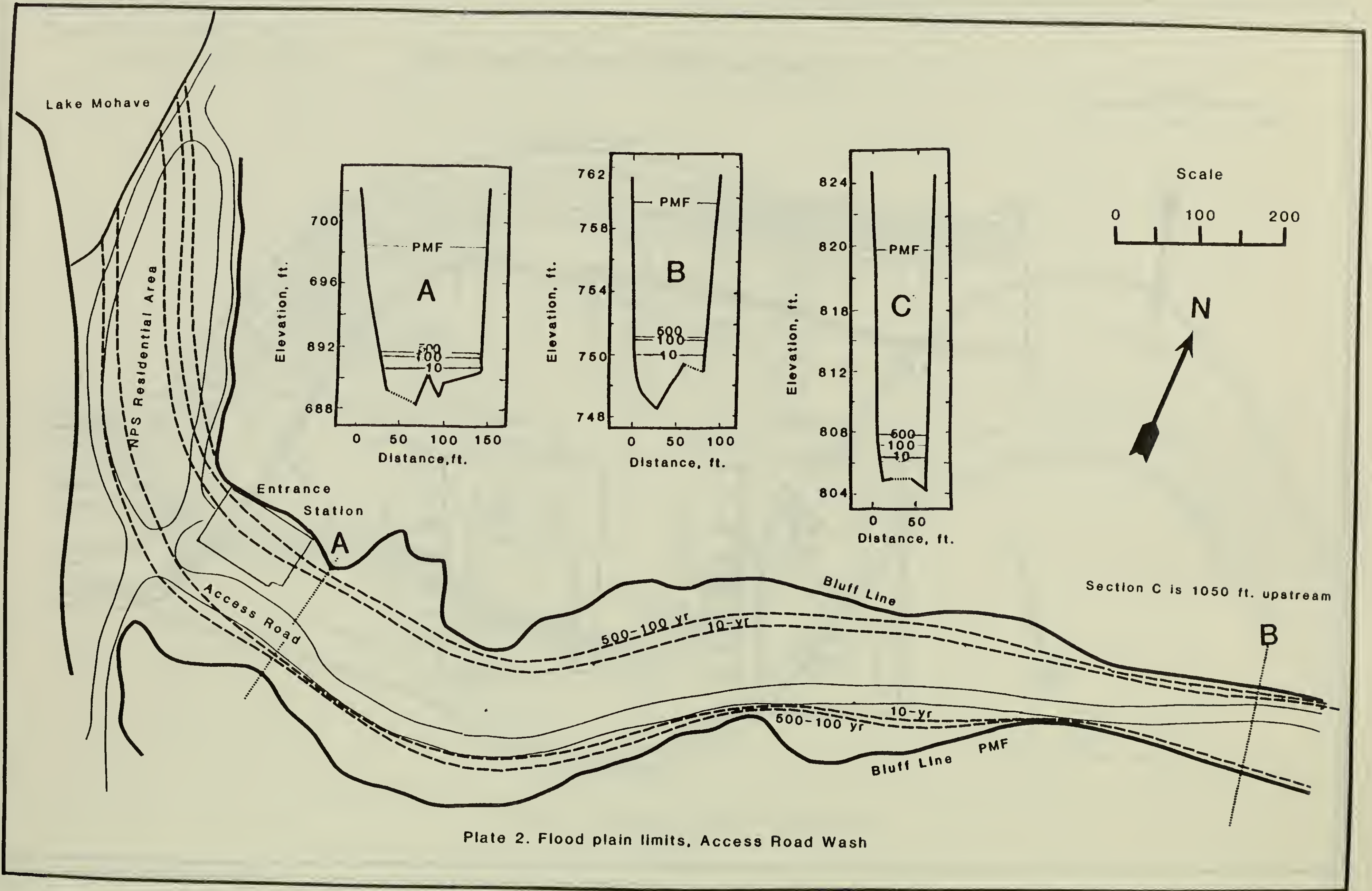


Plate 2. Flood plain limits, Access Road Wash

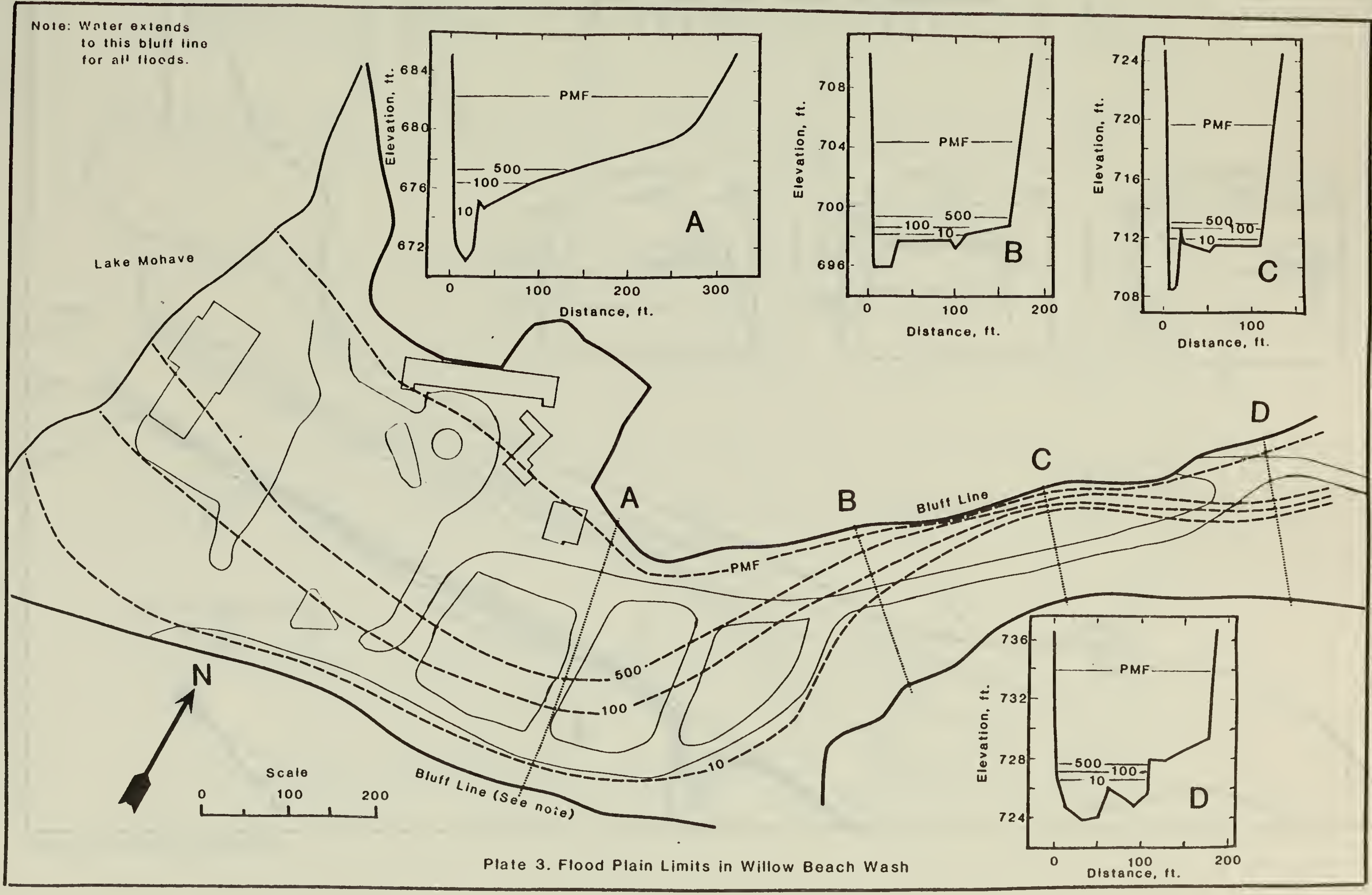


Plate 3. Flood Plain Limits in Willow Beach Wash

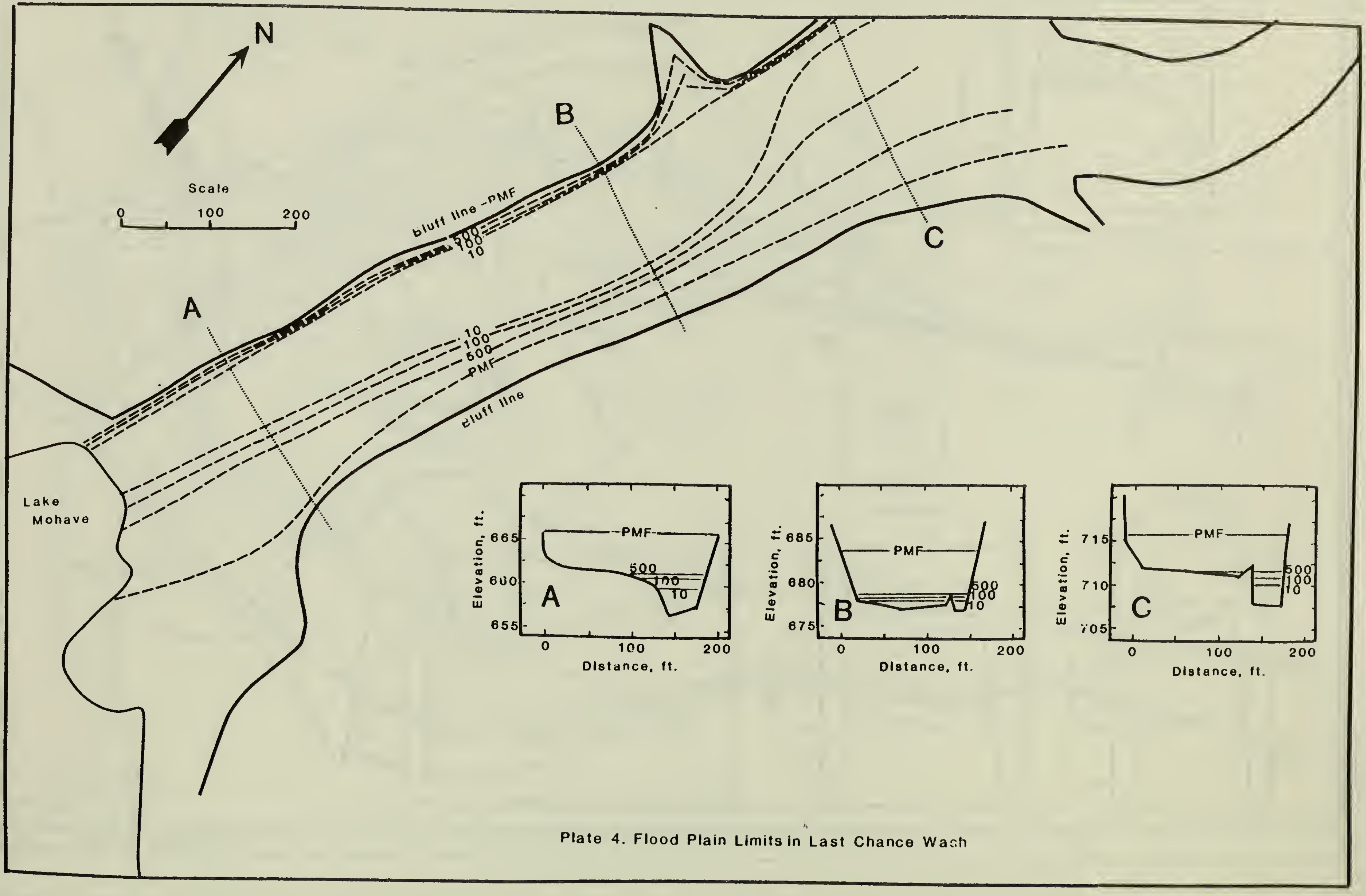


Plate 4. Flood Plain Limits in Last Chance Wash

Table 10

Water Depths and Velocities

Return Period	SECTION A mean depth ft.	max depth ft.	mean velocity ft/sec	SECTION B mean depth ft.	max depth ft.	mean velocity ft/sec
Access Road Wash						
2	1.3	1.6	6.5	1.1	2.2	6.0
5	1.6	2.0	7.1	1.0	3.2	5.8
10	0.9	2.5	5.5	1.2	3.3	6.1
25	1.1	2.7	6.0	1.4	3.5	6.7
50	1.3	2.9	6.5	1.6	3.8	7.2
100	1.4	3.0	7.3	1.8	4.0	7.7
500	1.8	3.5	7.7	2.2	4.5	8.6
PMF	7.7	10.0	15.9	10.0	12.9	18.1
Willow Beach Wash						
2	2.2	2.6	8.4	0.8	2.1	5.0
5	2.7	3.0	9.4	0.8	2.4	5.0
10	2.9	3.4	9.6	0.8	2.5	5.2
25	3.2	4.0	10.2	1.0	2.6	5.7
50	2.2	5.0	8.5	1.2	2.8	6.2
100	1.9	5.5	8.3	1.3	3.0	6.6
500	1.8	6.3	7.6	1.7	3.3	7.7
PMF	4.5	11.2	12.0	6.5	8.3	14.6
Last Chance Wash						
2	1.3	1.9	3.2	0.5	1.4	5.8
5	1.6	2.2	7.3	0.9	1.6	5.1
10	1.8	2.5	7.4	1.0	1.7	5.0
25	2.1	2.7	8.3	1.2	1.8	5.6
50	2.4	3.1	8.6	1.3	1.9	6.1
100	2.0	4.5	8.4	1.4	2.0	7.0
500	2.3	4.5	8.6	1.8	2.6	7.3
PMF	5.2	9.2	14.8	6.4	7.2	18.6
Jumbo Wash						
2	0.7	4.5	4.9	1.9	4.5	7.8
5	0.8	4.6	5.1	1.7	5.0	7.5
10	1.0	4.9	5.7	1.2	5.6	6.2
25	1.3	5.2	6.5	1.5	5.9	6.9
50	1.6	5.5	7.3	1.8	6.4	7.7
100	2.1	6.0	8.2	2.3	6.9	8.6
500	3.0	7.0	9.9	3.0	8.1	9.9
PMF	12.4	19.6	20.0	13.0	18.2	20.5

Table 10 Continued:

Return Period	SECTION C mean depth ft.	SECTION C max. depth ft.	SECTION C mean velocity ft/sec	SECTION D mean depth ft.	SECTION D max. depth ft.	SECTION D mean velocity ft/sec
Access Road Wash						
2	1.1	1.0	6.0			
5	1.4	1.3	6.8			
10	1.6	1.5	7.1			
25	1.9	1.8	7.8			
50	2.1	2.2	8.3			
100	2.2	2.5	8.9			
500	3.0	3.0	9.8			
PMF	12.2	15.5	20.1			
Willow Beach Wash						
2	2.8	3.4	9.4	0.8	2.2	5.1
5	1.0	3.6	5.6	1.0	2.4	5.8
10	1.1	3.7	5.9	1.1	2.6	6.1
25	1.3	3.9	6.4	1.4	3.8	6.6
50	1.5	4.1	7.0	1.5	3.1	7.0
100	1.7	4.4	7.5	1.8	3.3	7.6
500	2.2	4.8	8.4	2.2	3.7	8.4
PMF	8.6	11.3	16.7	6.3	10.1	14.4
Last Chance Wash						
2	1.6	1.8	7.0			
5	2.0	2.2	8.5			
10	2.2	2.4	8.5			
25	2.8	2.9	8.7			
50	3.0	3.0	9.7			
100	3.0	3.1	12.8			
500	4.7	4.9	10.0			
PMF	4.6	8.3	17.4			
Jumbo Wash						
2	1.6	2.5	7.2	1.9	2.3	7.8
5	1.8	2.9	7.5	1.9	2.7	7.9
10	2.4	3.6	8.8	2.4	3.4	8.8
25	3.1	4.3	10.1	3.1	4.1	10.0
50	3.6	5.1	10.8	3.8	4.8	11.1
100	4.4	6.1	11.9	4.8	5.8	12.5
500	3.6	8.0	10.8	6.9	8.2	15.0
PMF	13.3	18.3	20.7	12.7	23.7	20.7

The present flood hazard is high for facilities as they presently exist at Willow Beach. If sediment or debris should block the existing channels, even relatively minor floods could innundate the campground or trailer area.

FLOOD HYDROGRAPHS

Design of reservoirs for flood reduction requires some knowledge of the shape of the flood hydrograph. Figures 5 through 8 present the hydrographs for the 10-yr and 100-yr events. Table 11 summarizes the volume of runoff in the maximum hour of each flood. In our analysis we place the maximum hour of rain near the middle of the six hour period. If we had chosen to place the maximum intensities earlier, the hydrographs would have risen more abruptly but the volume of runoff and peak flow would have been somewhat less. Any time sequence is possible.

Table 11

Runoff Volume During the Maximum Hour

Period	Access Rd.	Willow Bch.	L. Chance	Jumbo
2	23	29	25	57
5	35	41	36	82
10	42	49	42	113
25	53	62	54	133
50	64	66	59	174
100	82	95	83	247
PMF	955	866	819	4820

THE PHYSICAL IMPACT OF FLOOD WATERS

Table 12 presents values of the force (in pounds) exerted on a circular cylinder immersed in water depths from 1 to 5 feet flowing at velocities from 2 to 20 feet per second. Assuming that the cylinder approximates a human body, a healthy adult might be able to stand in water 5 feet deep with velocity of 2 fps resisting a force of twenty pounds. However, the effective weight of this person would be very small because of bouyancy. Quite probably this person would be pushed or floated downstream, but at 2 fps might be able to swim to safety. With a velocity of 3 fps an adult could withstand depths of three feet and at 5 fps a depth of possibly 2 feet. At greater depths or velocities as indicated by table entries below the line, chances of survival would rapidly approach zero. A rough rule of thumb is that a healthy human adult can withstand a combination of velocity and depth whose product is 10, i.e. 2 fps and 5 feet deep or 5 fps and 2 feet deep.

Children because of their small weight and height would have serious difficulty at depths or velocities much lower than those a adult might withstand. Handicapped persons or persons trapped in sleeping bags on the ground could experience great difficulty in a depth of one foot with relatively low velocities. It should be noted that the discussion above applies to largely sediment-free water. If the flow is carrying very large sediment loads chances of survival are sharply reduced.

Forces exerted on cars, trailers, and other structure are

much greater than those indicated in Table 12 because of the much greater area exposed to the flow combined with the fact that the drag force on a large flat plate would be nearly double that on a cylinder of the same projected area. Bouyant forces at depths of 2 or 3 feet are usually sufficient to float an automobile or trailer.

In the Willow Beach Washes, water velocities exceed 5 fps at nearly all cross-sections in the 2-year flood and maximum depths are greater than 2 feet in most cross sections. Consequently, even the smaller floods would definitely be hazardous for children or physically handicapped if they were caught unexpectedly in the flow. For larger floods covering all or most of the canyon bottom the hazards are very great.

Table 12

Force in pounds exerted on a cylinder one foot in diameter when immersed in flowing water.

Depth of immersion, ft.	1	2	3	4	5	
Velocity of flow, fps						Tolerable
2	4	8	12	16	20	
3	9	17	26	35	44	
5	24	48	73	97	121	Unsafe
10	97	194	291	388	485	
15	218	436	655	873	1091	
20	412	825	1237	1649	2062	

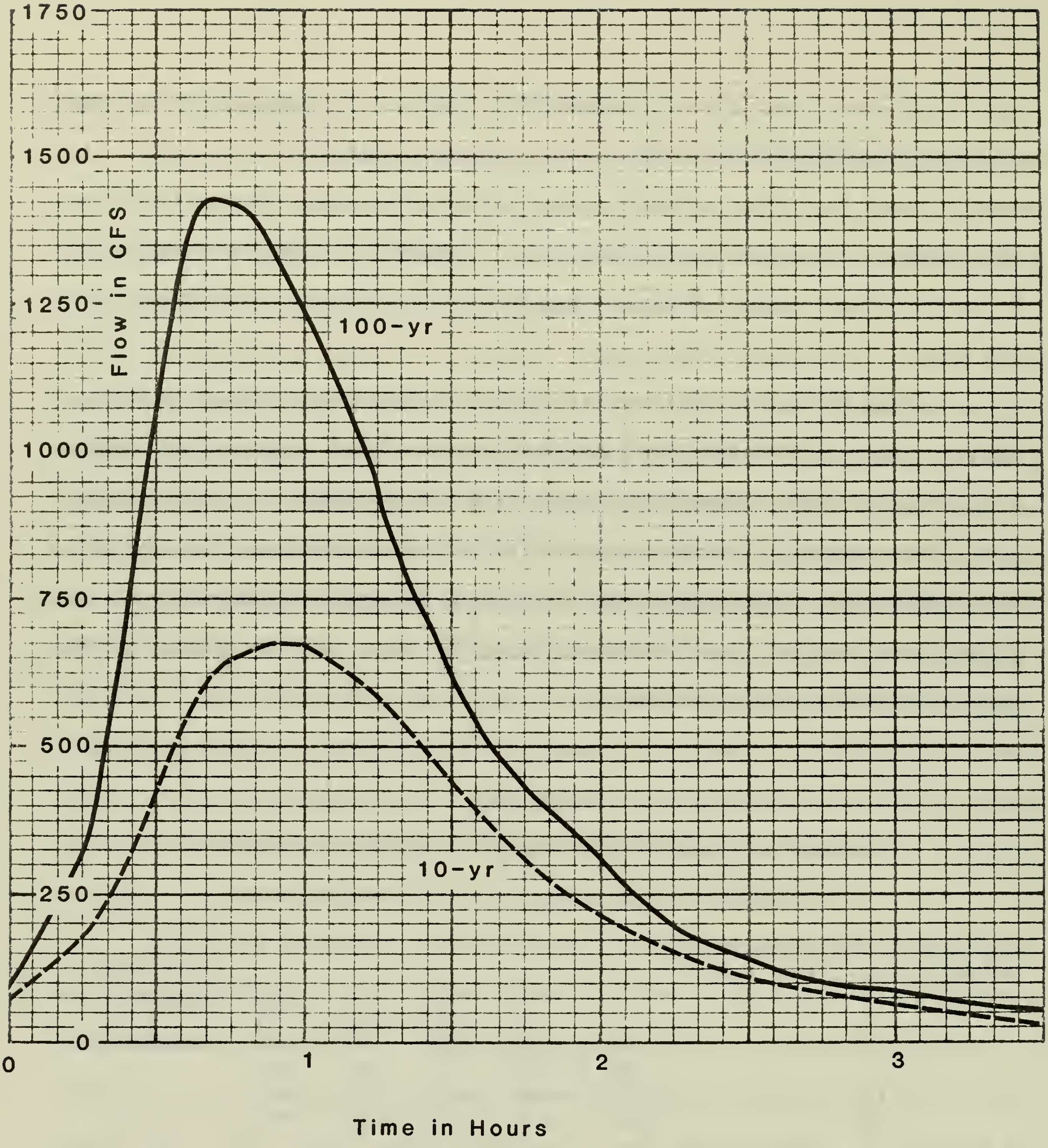


Fig. 5 Hydrographs of the 10- and 100- year floods on Willow Beach Wash

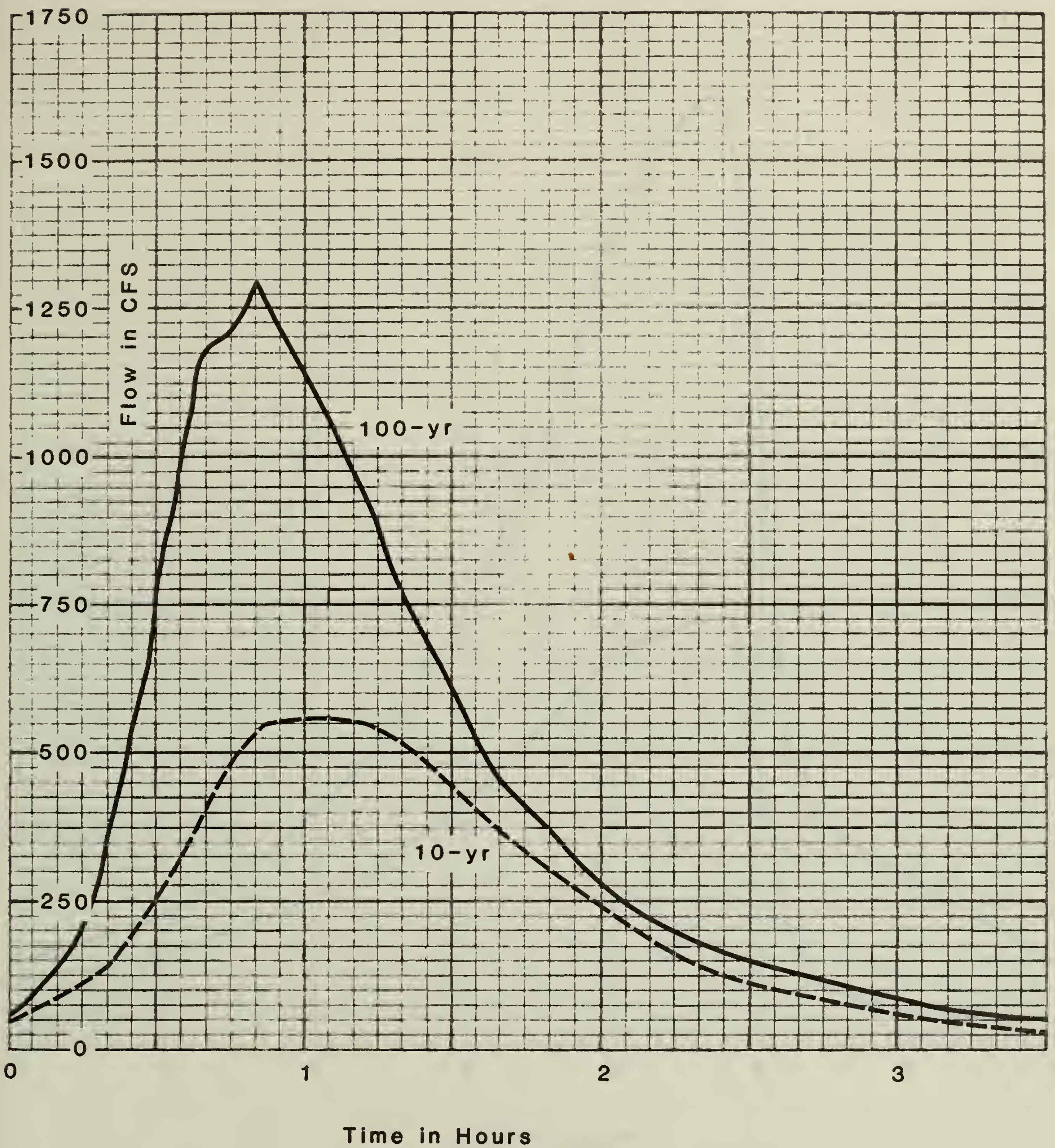


Fig. 6 Hydrographs of the 10- and 100-year floods on Last Chance Wash

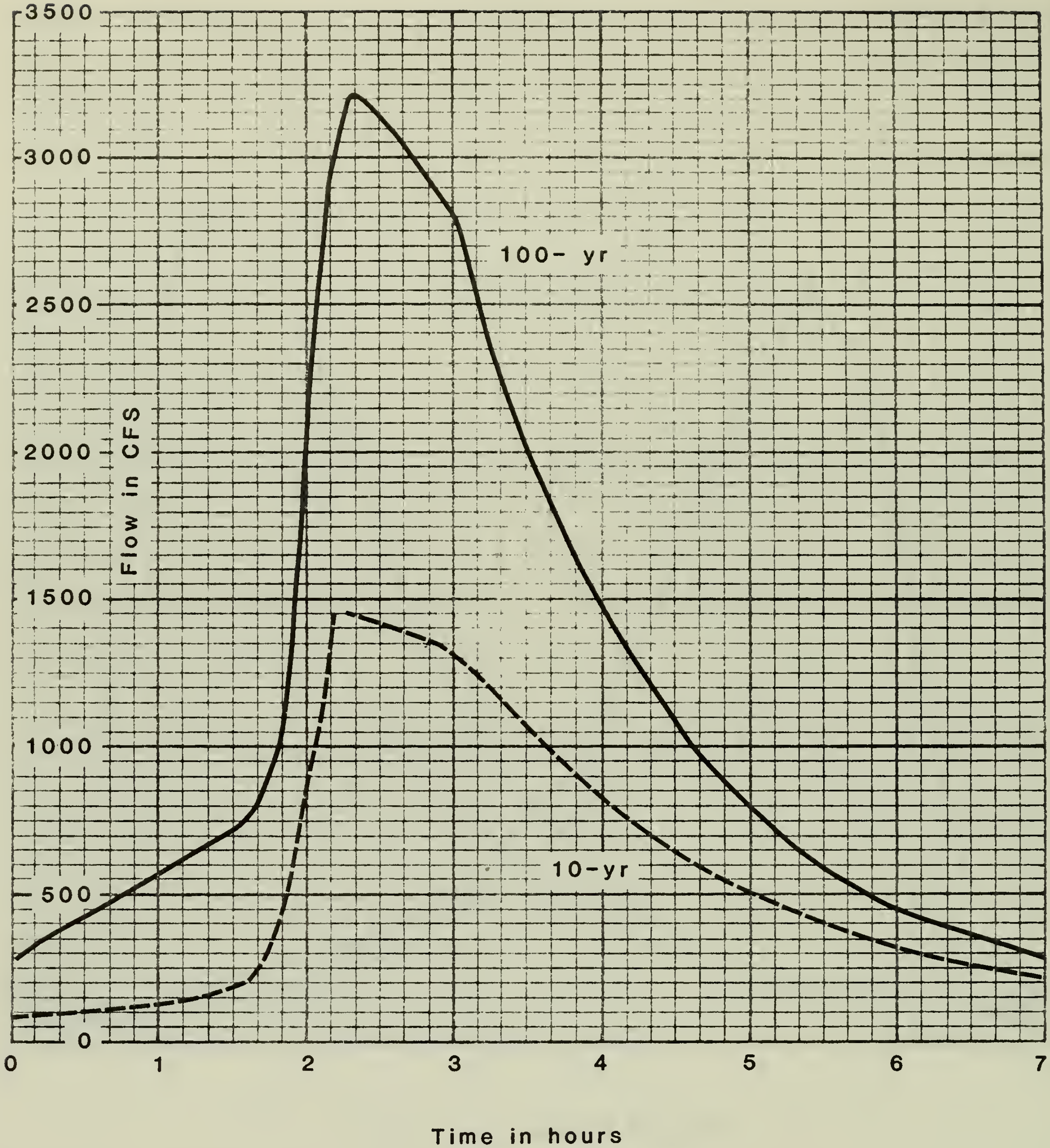


Fig. 7 Hydrographs of 10- and 100- year floods on Jumbo Wash

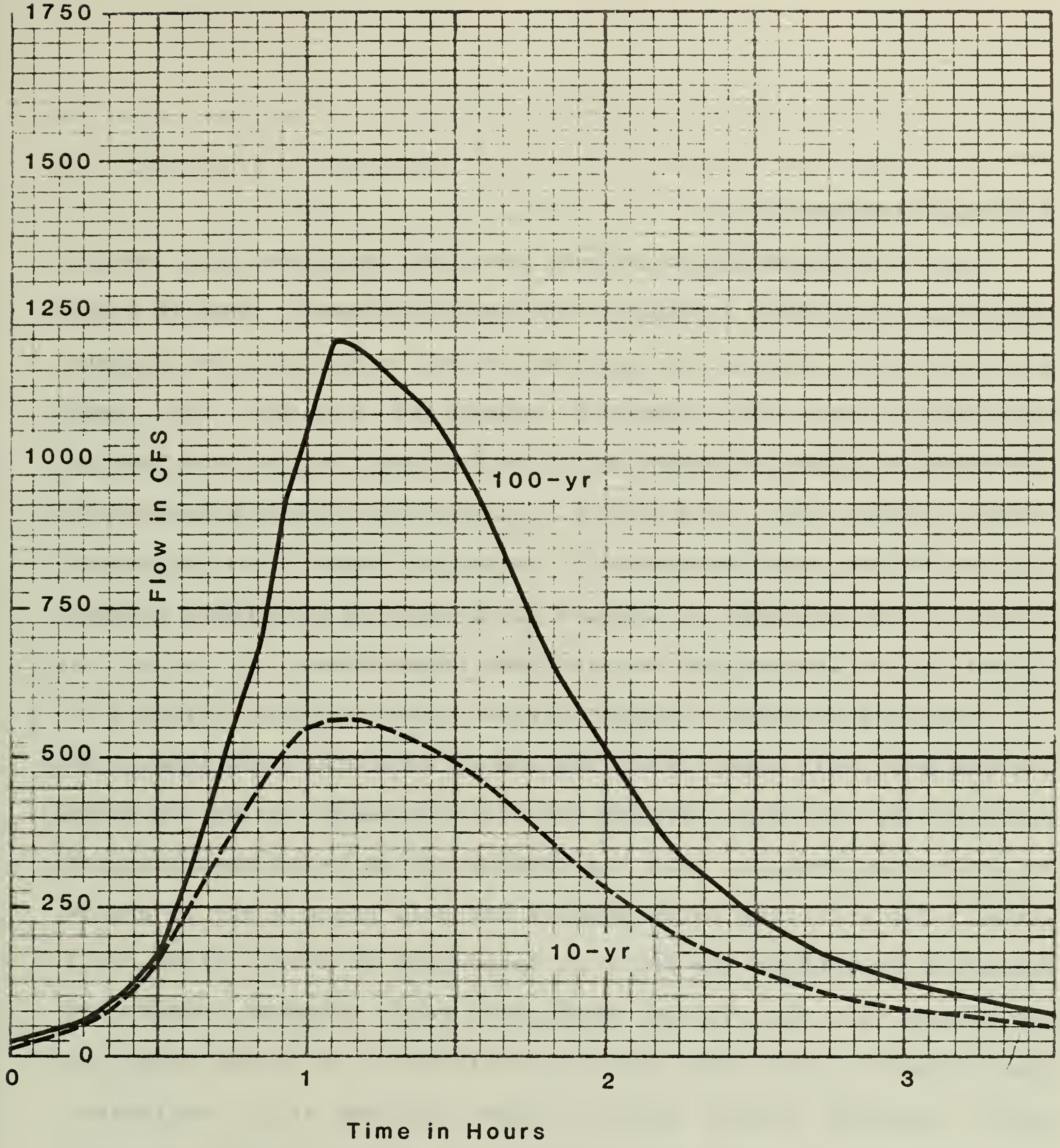


Fig. 8 Hydrographs of the 10- and 100-year floods on Access Road Wash

ALTERNATE METHODOLOGIES

Many other hydrologic methods have been employed for making estimates of flood frequency on ungaged streams. Many of these are obviously unsuited for application to the Willow Beach Washes. One method that has received widespread use is the Soil Conservation Service method which is decribed in the book "Design of Small Dams" [7]. This method, if used would be an alternate to the use of the modified Stanford Watershed Model, to calculate runoff. We elected to use the model because it permits greater direct use of information on watershed properties.

In the SCS method the appropriate runoff curve is selected on the basis of soil type, land use, and agricultural practices. As modified by the U.S. Forest Service, the selection may be based on soil-cover complex, with the closest option being sagebrush and grass. The resulting curve numbers for soil group B are 35 and 64 depending on whether the cover is rated good or poor. If we call the cover in the Willow Beach area good, computed runoffs and peaks would be much lower than our estimates. If it was rated as poor, computed runoff would be about the same as our estimates. Our assessment of the actual situation at Willow Beach is that most of the runoff arises from impervious area and that it is this fraction that governs the behavior of the catchments for the smaller floods.

For floods of the PMF magnitude infiltration capacity plays a

much more important role in determining the runoff volume from the very high rates of rainfall. The SCS method as described in reference [8] recommends the use of a curve number = 85 for a cover of brush, sage and grass of less than 50 percent density. This value is recommended for Zone II which includes all of the intermoutain west north of 35 degrees latitude. We find it difficult to believe the hydrologic conditions can be this uniform over such a large area. However, if CN = 85 were used the PMF estimates would be approximately 50 percent greater than those presented in this report. Thus, if the Curve Number approach were utilized the apparent hazard in the Willow Beach Washes would be even higher than we estimate.

The U.S. Geological Survey has prepared reports similar to Reference 2 for most states. Because these reports use geographic locations as an element of the analysis, only the report for Arizona is relevant for Willow Beach.

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